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FUNDAMENTAL CONCEPTS OF RECTANGULAR SETTLING TANKS

Alfred C. Ingersoll,[1] A.M. ASCE, Jack E. McKee,[2] M. ASCE, and Norman H. Brooks,[3] J.M. ASCE

SYNOPSIS

This investigation of fundamental theories of sedimentation, scour by turbulent eddies, inlet and outlet disturbances, and sludge thickening phenomena indicates that settling tanks should be designed on the basis of surface area and that rectangular settling tanks should be long and narrow. The minimum depth of such tanks is limited not by bed-load movement resulting from direct shear, as previously supposed, but by scour resulting from turbulent eddies in accordance with the suspended load equation. Tanks can be made relatively shallow if the deposited sludge is protected from resuspension by means of multiple inclined baffles. These baffles also serve to produce thicker sludge.

The conventional measure of efficiency of settling tanks is highly inadequate. In its place a new rational measure, called the overflow residual efficiency, has been developed theoretically.

I. INTRODUCTION

The primary function of a settling tank is to separate settleable and floatable solids from the liquid in which they have become suspended. In addition, settling tanks are expected frequently to perform another major role, namely to separate liquid from the settled solids, a process known as "sludge thickening." A well-designed settling tank should be able to accomplish both of these functions simultaneously at optimum efficiency, with maximum economy of construction and minimum costs for operation and maintenance.

Like many other phases of engineering, however, the design of settling tanks has been governed largely by empirical rules rather than by fundamental considerations. Despite the excellent theoretical treatises by Hazen[4] and Camp[5] showing that sedimentation is primarily a function of the settling velocity of discrete particles and largely independent of depth, the design of many settling tanks for water and sewage works is still being based on detention period and rules of thumb.

It is one of the purposes of this paper, therefore, briefly to reiterate the fundamental concepts of sedimentation, especially as they relate to the shape

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4. "On Sedimentation" by Allen Hazen, Trans. ASCE, Vol 53 (1904), p. 63.
5. "Sedimentation and the Design of Settling Tanks" by Thomas R. Camp, Trans. ASCE, Vol. 111 (1946), p. 895.

of rectangular settling tanks and the length: width: depth ratios. Measures of efficiency are analyzed critically and a new method is proposed for comparing the operating efficiencies of settling tanks. In addition, the paper presents recently acquired empirical data that extend the basic knowledge and it propounds some theoretical aspects that have received little emphasis in civil-engineering literature. Sludge thickening is discussed in relation to the area and depth of settling tanks, and a method is proposed to improve thickening simultaneously with settling efficiency. Finally, it is the intent of the authors to suggest areas in which research may be conducted in order to further the understanding of this important phase of hydraulic design. Owing to limitations of space, all aspects of sedimentation and the design of rectangular settling tanks cannot be included in this paper, and in many instances such aspects are treated merely by reference to other detailed and authoritative papers.

II. Resume of Fundamental Concepts

The settling of discrete non-flocculent particles in a quiescent liquid, as reported by many investigators, is described in recent textbooks of fluid mechanics. In very weak concentrations, the particles settle individually, but in thicker suspensions the settling velocities are reduced materially by collisions and by interference of the velocity fields of particles, combined with the upward movement of displaced liquid. McNown and Lin[6] show that settling velocities in a suspension of 1000 ppm by volume are decreased by as much as 13%, while at a volumetric concentration of 10,000 ppm the settling velocity is decreased by as much as 25%. In a quiescent suspension, the concentration may be weak in the top layers but it will become progressively more concentrated with increasing depth from the surface. When the concentration becomes so thick that numerous particles adhere to form a continuous honeycombed structure, the resulting sludge blanket will subside initially in accordance with laws of consolidation. (See Section VII.) In flocculent suspensions, two or more particles may coalesce to form a larger agglomerate which will probably settle at an increased rate.

Hazen[4] and Camp[5] analyzed the settling of discrete particles in a so-called "ideal basin," for which the following assumptions apply: (a) the direction of flow is horizontal and the velocity is uniform in all parts of the settling zone, (b) the concentration of suspended particles of each size is uniform over the depth at the inlet end of the settling zone, and (c) particles reaching the bottom remain fixed. For these conditions it is readily demonstrated that the settling velocity of a particle that just falls through the depth during its theoretical period of detention is equal to the discharge per unit of surface area, known as the "overflow rate." All particles with settling velocities v, greater than the overflow rate v_o will be separated from the suspension while particles with $v < v_o$ will be removed in the ratio $\frac{v}{v_o}$. Because Hazen was the first to utilize it in the sedimentation field to which he contributed so greatly, the authors propose that this ratio be named the "Hazen number;" thus $N_H = \frac{v}{v_o}$. The use and significance of the Hazen number are explained in Section III.

6. "Sediment Concentration and Fall Velocity" by John S. McNown and Pin-Nam Lin, Proceedings of the Second Midwestern Conference on Fluid Mechanics (Ohio State Univ.) (1952) p. 401. Also see Trans. ASCE, Vol. 117 (1952), p. 440.

Sedimentation in an "ideal" tank is independent of depth, being a function merely of the discharge and surface area. Were it not for factors that militate against the assumptions of an ideal basin and interfere with the theoretical conclusions, settling tanks could be made infinitely shallow. In real basins, however, the theory is modified by the effects of flocculation, turbulence, scour, density currents, inlet and outlet disturbances giving rise to mixing and short-circuiting, and the movement of cleaning mechanisms. In describing and evaluating these effects and in prescribing for their amelioration, Camp[5] indicates that:

1. Settling tanks should be designed on the basis of overflow rate, i.e. the settling velocity of the smallest particle theoretically to be 100% removed.
2. Detention periods, per se, are immaterial; in fact, for sewage treatment long detention periods may be deleterious.
3. Tanks should be no deeper than will be required to prevent scour and to accommodate cleaning mechanisms.
4. Tanks should be long and narrow to minimize the effects of inlet and outlet disturbances, cross winds, density currents, and longitudinal mixing.

Despite the apparent simplicity of the theories of sedimentation and the clear relation of overflow rates to removal ratios, attempts to compare the performances of settling tanks in water or sewage works have been disappointing. There appears to be little or no correlation in actual operating records between removal efficiencies and overflow rates or detention periods. Basins that are poorly designed on the basis of theory sometimes appear to perform better than well-designed tanks. In an attempt to evaluate the performance of settling tanks and to find parameters by which tanks may be compared logically, the following analysis of measures of efficiency is advanced.

III. Measures of Efficiency

A true measure of the efficiency of a settling tank should satisfy two requirements: (1) it should be independent of the characteristics of the suspension, and thus it should reflect directly the adequacy of the hydraulic design and operation of the tank, and (2) for an ideal tank, the efficiency should be 100%.

The conventional measure of efficiency, i.e. the total per cent reduction of suspended solids, does not satisfy these requirements. It is obvious that any given tank with a given overflow rate will remove a far larger proportion of the solids from a suspension of coarse particles than from a suspension of very fine particles. For an ideal tank, furthermore, the total per cent reduction of suspended solids is not 100%, but a variable quantity depending on the size distribution of the material in suspension. Hence, if a chosen measure of efficiency is a function largely of the size distribution and is not 100% even for an ideal tank, then it is a hopeless basis for comparison of different tanks unless the overflow rates and the suspensions to be settled are identical.

The effects of various size distributions on total removal can best be visualized by considering the frequency distribution curve for the settling velocities of the particles. Such a distribution curve may be derived from a cumulative curve as illustrated by Fig. 1. A hypothetical raw (influent) distribution curve, $f_r = f_r(v)$, and the corresponding distribution curve for the effluent from an ideal tank, $f_i = f_i(v)$, are shown in Fig. 2. By definition $f_r \Delta v$ is the fraction of the total material (by weight) that has a settling velocity between $v - \frac{\Delta v}{2}$ and $v + \frac{\Delta v}{2}$. The total area under the f_r curve is unity and corresponds to the total concentration of solids.

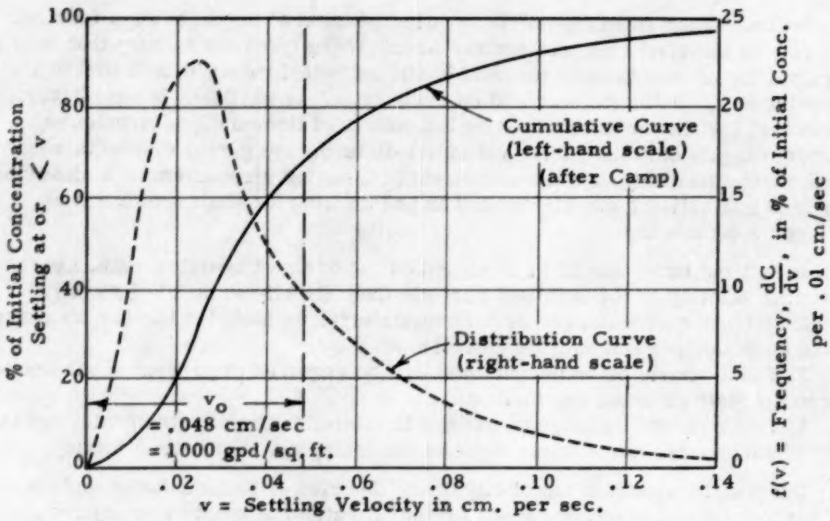

Fig. 1. Concentration as a Function of Settling Velocity

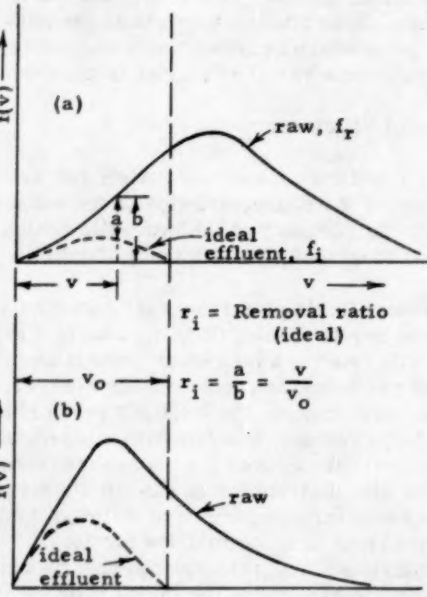

Fig. 2. Influent and Effluent Relationships For Two Suspensions in an Ideal Tank

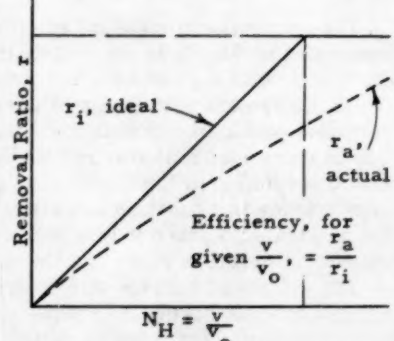

Fig. 3. Removal Ratio as a Function of Hazen Number

If r_i denotes the removal ratio in an ideal tank (or simply the "ideal removal ratio") for a particular size of particle, then

$$r_i = N_H \quad \text{for } N_H < 1$$

$$\text{and} \quad r_i = 1 \quad \text{for } N_H \geqslant 1$$

where N_H, the Hazen number, is defined in Section II as $\frac{v}{v_o}$. The ideal removal ratio is shown graphically as a function of N_H in Fig. 3. The distribution curve f_i for the ideal effluent may then be given by the formula

$$f_i = f_r (1 - r_i) \quad \ldots\ldots (1)$$

Referring again to Fig. 2 note that for $v > v_o$, f_i is always 0, indicating complete removal, whereas for $v < v_o$, the removal is proportional to $\frac{v}{v_o}$.

The total ideal removal, R_i, is defined as the weighted average of the individual removals, r_i, i.e.

$$R_i = \int_o^\infty r_i f_r \, dv \quad \ldots\ldots (2)$$

Because the total ideal removal ratio is also the area between the f_r and the f_i curves, divided by the area under the f_r curve, R_i can also be written as:

$$R_i = \frac{\int_o^\infty f_r \, dv - \int_o^\infty f_i \, dv}{\int_o^\infty f_r \, dv}$$

$$\text{since} \int_o^\infty f_r \, dv = 1, \quad R_i = 1 - \int_o^\infty f_i \, dv \quad \ldots\ldots (3)$$

Now, comparing Fig. 2a with Fig. 2b it is quite apparent that the total ideal removal R_i in case (a) is much larger than in case (b). Although the individual ideal removal ratios r_i are identical in (a) and (b) for each value of v, case (b) is a finer suspension, and hence the sizes for which the removal is incomplete carry more weight in the integration.

Since any real tank has some turbulence, short-circuiting and possible scour, the actual removal ratio, r_a, will probably fall short of the ideal removal ratio r_i, as illustrated in Fig. 3. If the suspended material flocculates as it settles, it is even possible for r_a to exceed r_i. The actual effluent distribution curve f_a may now be written in the same manner as the ideal case, as

$$f_a = f_r (1 - r_a) \quad \ldots\ldots (4)$$

A hypothetical actual effluent curve f_a is shown in Fig. 4. In contrast to f_i for the ideal effluent, f_a does not equal 0 for all values of $v > v_o$.

The total actual removal ratio R_a is similar to R_i and is defined as:

$$R_a = \int_0^\infty r_a f_r \, dv$$

or

$$R_a = 1 - \int_0^\infty f_a \, dv \qquad \ldots\ldots (5)$$

This is the conventional measure of performance. Like R_i, unfortunately, its magnitude is affected by the raw distribution. Thus, without knowing the settling-velocity distribution of the suspended matter in the influent, and hence, not knowing R_i either, it is impossible to compare quantitatively the performances of different settling tanks merely on the basis of the total removal R_a and the overflow rate v_o.

Efficiencies

For any particle size the efficiency, e, may be defined as the ratio of actual to ideal removal ratios,

$$e = \frac{r_a}{r_i} \qquad \ldots\ldots (6)$$

This is an efficiency in the true sense because it is a measure of what the tank accomplishes relative to what the <u>same</u> tank should <u>ideally</u> accomplish, whether r_i be large or small. The amount that e falls short of unity is entirely the net result of turbulence, short-circuiting, and flocculation. If flocculation is significant in increasing the removal of particles for which $N_H < 1$ then it is possible for the efficiency at a given particle size to exceed unity. It is probable that the efficiency will vary somewhat with the Hazen number. As N_H becomes very large, good removal is assured in spite of short-circuiting and turbulence and the efficiency, e, is bound to approach unity.

It is logical now to define a total or integrated efficiency E as:

$$E = \frac{R_a}{R_i} = \frac{\int_0^\infty r_a f_r \, dv}{\int_0^\infty r_i f_r \, dv} \qquad \ldots\ldots (7)$$

In a sense this is a weighted average efficiency, since it can also be written:

$$E = \frac{\int_0^\infty e \, r_i f_r \, dv}{\int_0^\infty r_i f_r \, dv} \qquad \ldots\ldots (8)$$

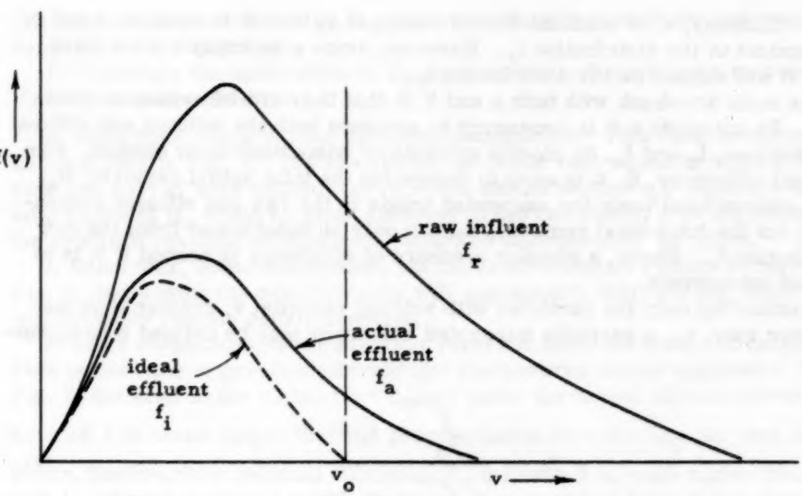

Fig. 4. Actual Performance Compared With Ideal Basin

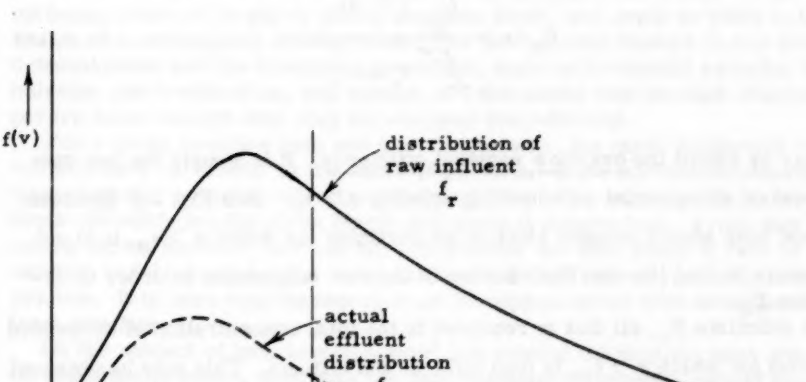

Fig. 5. Hypothetical Actual Effluent Distribution

If the efficiency, e, is constant for all values of v, then E is equal to e and is independent of the distribution f_r. However, since e probably varies somewhat, E will depend on the distribution f_r.

The main drawback with both e and E is that they are laborious to determine. To calculate e it is necessary to measure both the influent and effluent distributions, f_r and f_a, by pipette analysis or some equivalent method. For the total efficiency, E, it is easy to determine the total actual removal, R_a, using conventional tests for suspended solids in the raw and effluent suspensions; but the total ideal removal, R_i, can only be determined from the raw distribution f_r. Hence, a simpler measure of efficiency is needed if it is to be used extensively.

Considering only the particles with settling velocity, v, greater than the overflow rate, v_o, a partially integrated efficiency may be defined conveniently as:

$$E_o = \frac{\int_{v_o}^{\infty} r_a f_r \, dv}{\int_{v_o}^{\infty} r_i f_r \, dv} \qquad \ldots\ldots (9)$$

Thus, E_o is the area between the f_r and f_a curves for $v > v_o$ divided by the area under the f_r curve for $v > v_o$. For $v > v_o$, however, $r_i \equiv 1$ and $r_a = e$. Hence,

$$E_o = \frac{\int_{v_o}^{\infty} e f_r \, dv}{\int_{v_o}^{\infty} f_r \, dv} \qquad \ldots\ldots (10)$$

E_o may be called the <u>overflow residual efficiency</u>. It is simply the per cent removal of all material with settling velocity $v > v_o$. (See Fig. 5.) Because an ideal tank should remove 100% of all particles for which $v > v_o$, it is not necessary to find the size distribution of the raw suspension in order to determine E_o.

To calculate E_o, all that is required is the total concentration of suspended material for which $v > v_o$, in both influent and effluent. This may be obtained for a sample of influent or effluent by separating the large from the small particles in an upward-flow device where the backflow velocity is equal to the overflow rate v_o for the tank. The concentration of suspended matter in the measured volume of liquid remaining in the apparatus may then be determined for each sample. Such an apparatus might be developed, standardized, and made a part of sanitary engineering laboratories.

The overflow residual efficiency, E_o, is a relatively simple measure of efficiency to determine, but unfortunately it is still somewhat dependent on the size distribution of the material, inasmuch as the efficiency, e, depends on v. Nevertheless, since it does have the value 100% for any ideal tank, it should give a reasonably valid comparison between tanks of different design, whereas

the total actual removal ratio, R_a, as conventionally used is an inadequate basis for such comparison.

To illustrate the application of E_o as a measure of performance, consider the following combinations:

1. Same tank, same v_o, but different suspensions. This case is shown for an ideal tank in Fig. 2. The results for an actual tank would be similar except that part of each effluent curve would extend beyond v_o. Although the total removal R_a would be quite different in the two cases, it is expected that the overflow residual efficiency E_o would be substantially the same.

2. Same tank, same suspension, but different overflow rates. As shown in Fig. 6, decreasing the overflow rate will appreciably increase R_a, while E_o will probably remain nearly constant.

3. Same suspension, same overflow rate, but different design of tanks. This is the case of greatest interest and controversy among engineers. In Fig. 7, the area to the right of $v = v_o$ and under the actual effluent curve f_{a_1} for tank 1 is much larger than the corresponding area for f_{a_2}, for tank 2. Hence, the overflow residual efficiency E_o for tank 2 is much higher than for tank 1. Therefore tank 2 performs better than tank 1 at least for particles with $v > v_o$, and most probably for all $v < v_o$ as well.

Let us consider next how some of the factors of design can be used to improve the efficiency and performance of settling tanks.

IV. The Shape of Rectangular Tanks

On the basis of the theory of "ideal" settling tanks, one cannot define the optimum ratios of length to width, length to depth, and depth to width in the design of a rectangular settling tank. The fundamental factors in any such determination are the kinematic quantities, such as horizontal velocity, turbulence, short-circuiting, and mixing. To the extent that the tank dimensions govern these factors they may be analyzed theoretically.

For a given overflow rate and plan dimensions, the mean horizontal velocity will vary inversely as the depth of the tank. If the horizontal velocity is limited by the scouring velocity of the particles being settled, the minimum depth allowable for the given length and width is determined. From this reasoning alone, however, one can not tell whether the best shape of tank is long, narrow, and deep or, for the same surface area and velocity, short, wide, and shallow. It is here that the theory must be supplemented with observations from practice.

On the subject of tank length, Camp[5] has pointed out that the inlet and outlet disturbance zones, even with the best designed structures, extend into the tank for a distance at least equal to the depth from each end. This means that a long narrow tank will lose a minimum of its surface area to the unproductive inlet and outlet disturbance zones. He indicates that the scouring velocity establishes a minimum for the depth of a tank of fixed width, but this dimension is rather more frequently controlled by the minimum requirement for the installation of a sludge-scraping mechanism. Commonly accepted minimum depths for mechanically cleaned tanks are between 5 and 8 feet.

For the settling of flocculent suspensions, Camp has given a detailed method for determining the exact dimensions of tanks of different types. In lieu of a similar theoretical approach in this paper, the authors have found it instructive to make some observations on the results of two sets of laboratory model investigations.

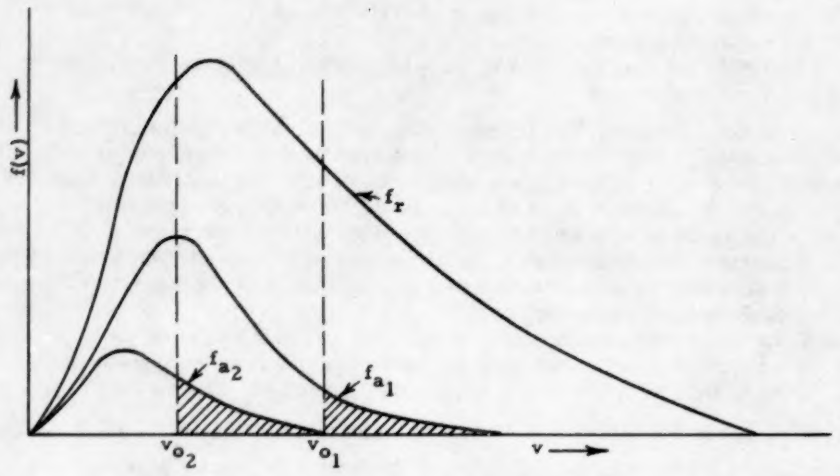

Fig. 6. Same Tank, Same Suspension, But Different Overflow Rates

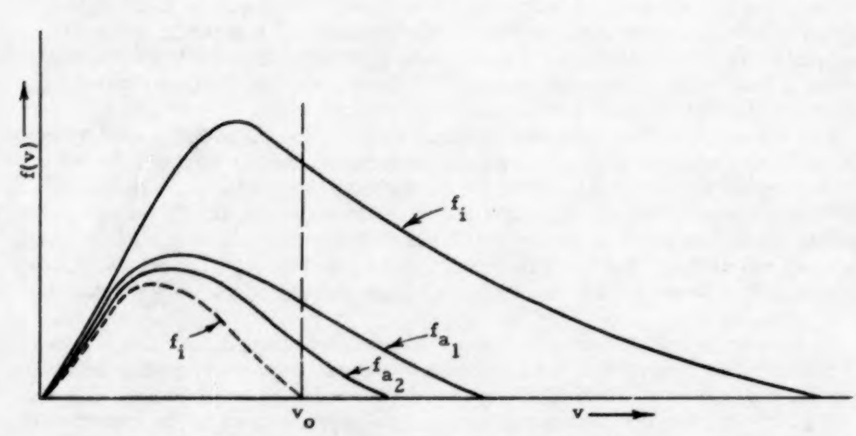

Fig. 7. Same Suspension and Same Overflow Rate, But Different Design of Tanks

Experimental Results

At the University of Wisconsin[7] a model basin 10 ft long by 5 ft wide by 2 ft deep was tested with a 200-ppm suspension of microcrystalline wax spheres of specific gravity 0.92 and approximate diameter 0.02 cm. The basin was supplied with a vertical slotted inlet baffle obstructing 96% of the free area, and a simple surface retention baffle followed by an overflow weir at the outlet. The tank was so constructed that the length could be made 5 or 10 ft, the width 2 or 5 ft, and depth 0.2, 0.4, 0.6, 0.8, 1.0, 1.5, or 2.0 ft.

A similar tank was constructed at Purdue University,[8] of fixed length 11.5 ft, adjustable widths of 2.0, 4.0, and 6.0 ft, and adjustable depths of 1.1 and 2.1 ft. The influent passed through two horizontal slotted baffles, obstructing 92% of the free area, while the effluent passed over a simple weir. The tank was tested with a suspension of approximately 200 ppm of diatomaceous silica. Separate tests, run in quiescent cylinders concurrently with the tank tests, afforded some information on the settling analysis of the suspension.

The most important difference between the two sets of tests was that the silica settled downward, and was therefore subject to scour along the bed, whereas the wax moved upward to the free surface, where there was presumably no shear or scour. Actually, however, the surface of the wax tank soon became covered with a stagnant film of wax, held by the retention baffle, and the wax rising to the under side of this layer must have been subject to much the same shear forces as the downward-settling silica. It is not surprising, therefore, to see that both sets of tests yielded results that are closely comparable.

The data from the 37 wax tests and the 35 silica tests have been plotted together in Fig. 8 to show the total actual removal in per cent as a function of the Hazen number. It is evident that the disparity in the results of either the wax or the silica tests attributable to changes in shape of the tanks is far greater than the difference in the results from one set of tests to the other. Although a consistent difference on account of the aforementioned condition of reduced surface shear for the wax might logically be expected, it is not apparent.

The Hazen number was computed for the median particle of the suspension in both cases. The distribution of rising velocities for wax spheres was determined by a sieve analysis of the wax, and values of rising velocity were computed by Stokes' and Newton's laws. The distribution of the silica settling velocities was determined by quiescent settling tube tests.

Although it is difficult to draw any firm conclusions from examining Fig. 8, a few salient features may be noted. At a value of $N_H = 0.38$, the wax tests in a tank of dimensions 8.9 ft long x 2.0 ft wide x 1.5 ft deep produced a removal of 36% which is very close to that which might be expected of an "ideal tank" receiving a suspension of uniform particles, all the same size as the median particle of the true suspension. Varying the depth from 1.5 or 2.0, down to 1.0, 0.8, and 0.6 ft all produced only negligible changes in removal, well within the range of experimental errors. When the depth was reduced to 0.4 ft,

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7. "Hydraulic Characteristics of Gravity Type Oil-Water Separators" by Gerard A. Rohlich, Proc. Amer. Petrol. Inst., Vol. 31M(III)(1951) p. 63.
8. "An Investigation of the Effect of Varying the Width and Depth of a Sedimentation Tank" Purdue Univ. Progress Report No. 3 of Project No. RG 1785C, of the Div. of Research Grants, National Institute of Health (1953).

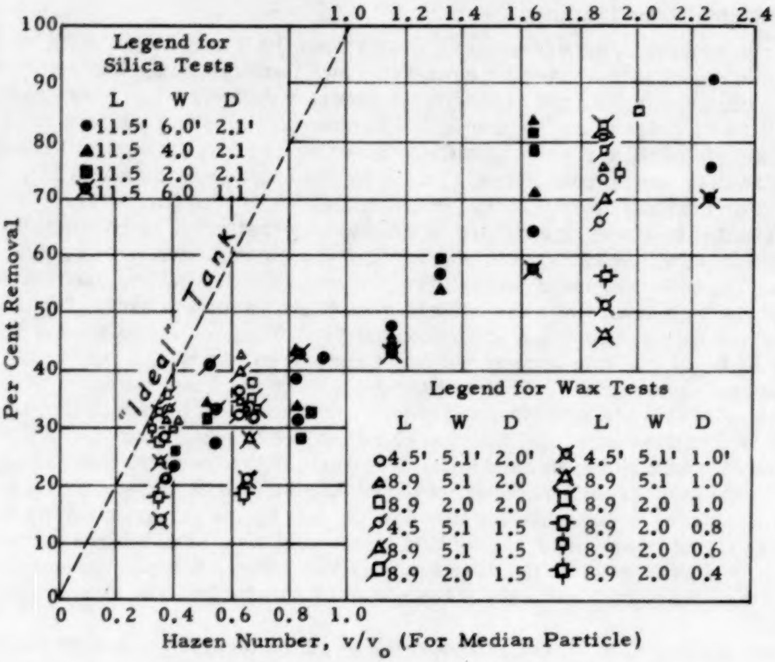

Fig. 8. The Effect of Length, Width, and Depth on the Removal of Silica and Wax in Model Basins

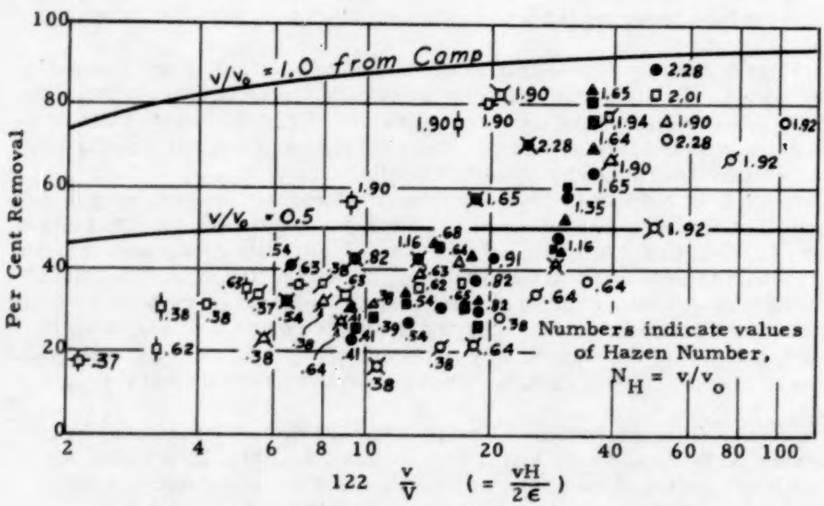

Fig. 9. Effect of Turbulence on Wax and Silica

however, the removal suffered a marked drop to 16%. This identical trend was observed at the other two values of Hazen number tested with wax. Even poorer than the long shallow tank was the "square" tank, 4.5 x 5.1, x 1.0 ft. With this shape the removal was significantly improved as the depth was increased to 1.5 and 2.0 ft, at all values of N_H. The long, narrow tanks appear to lead the field, both in the wax and the silica tests. The silica data from the shallow tank indicate a decreased removal at high values of Hazen number, but this tank shows comparatively good removal for values of N_H less than 1.0.

It is evident from the results of both sets of tests that the removals obtained were generally far short of the line representing the ideal tank. It is natural to inquire whether this may be explained on the basis of the turbulent diffusion theory forwarded by Dobbins[9] and discussed by Camp. The wax and silica data have been replotted on Fig. 9 utilizing exactly the same coordinates employed by Camp in Fig. 14 of his paper.[5] Beside each plotted point appears the value of $N_H = v/v_o$. The two lines for Hazen numbers of 1.0 and 0.5 have been traced from Camp's curve. Both wax and silica tests, but especially the former, show a decrease in removal with increase in horizontal velocity. For example, in the 8.9 ft long by 2.0 ft wide tank with wax, doubling the horizontal velocity (or halving the abscissa of Fig. 9, as from 20 to 10) reduces the removal from about 80% to about 60%, for a constant Hazen number of 1.9. In the 11.5 x 6 ft tank with silica, the reduction in abscissa from 51 to 27 results in a decrease in removal from about 80% to about 70%, for constant $N_H = 2.28$. For wax tests in the shorter tank, a reduction in abscissa from 102 to 51 results in a decrease in removal from 74% to 51%, for $N_H = 1.92$.

When these marked rates of decreasing removal with increasing velocity are compared with the gentle slopes of the theoretical lines, it becomes clear that the effect of turbulent diffusion may be greater than predicted from theory, or that other effects such as short-circuiting or scour are operating.

In the case of the tank at Wisconsin, separate studies of short-circuiting were conducted by the dispersion technique. (See Section VI.) These tests, reported by Rohlich[7] and by Ingersoll,[10] indicate that the effective overflow rate varied from about 1.10 to 1.05 times the observed overflow rate, over the same range of horizontal velocities mentioned above. This difference, of course, is not great enough to indict short-circuiting for the discrepancy between the results of theory and experiment. A more plausible explanation exists in a consideration of scour.

Scour by Turbulent Eddies

Based on experiments by Shields and others relating to bed-load movement, Camp[5] derived a formula for the mean channel velocity, V_c, required to start motion of particles on the bottom of a settling tank. When applied to the foregoing experimental data from Wisconsin and Purdue this formula gives values of critical channel velocities, V_c, much greater than actual tank displacement velocities, V, which might lead one to conclude that scour was not a factor in those tests.

Camp's formula works well in determining the maximum permissible velocity for sand particles in a grit chamber but unfortunately it does not apply to

9. "Effect of Turbulence on Sedimentation" by William E. Dobbins, Trans. Am. Soc. C.E., Vol. 109 (1944) p. 629.
10. "Determination of Hydraulic Characteristics of Separating Chambers" by Alfred C. Ingersoll, Proc. Midwest Conf. on Fluid Dynamics, Univ. of Illinois (1950), Vol. 1, p. 389.

the fine, light, flocculent material that forms the upper layer of the sludge blanket in a water or sewage tank. From Fig. 11 of Camp's paper, it may be seen that the experimental points plotted by Shields for the critical shear for initiation of bed movement fall in the range of $0.15 < \frac{d}{\delta'} < 40$, or in terms of the bed Reynolds number, $\frac{d\sqrt{\tau_o/\rho}}{\nu} = 11.6\,\frac{d}{\delta'}$, in the range $1.7 < \frac{d\sqrt{\tau_o/\rho}}{\nu} < 450$. For typical sewage solids in a settling tank, the values of $\frac{d\sqrt{\tau_o/\rho}}{\nu}$ are much smaller than 1.7, so that Shields' results cannot be applied. For example, if we assume that V = 3 ft/min and the Darcy friction factor f = .025, then $\sqrt{\frac{\tau_o}{\rho}} = \sqrt{\frac{f}{8}}\,V = .0028$ ft/sec; taking d = 0.1 mm and $= 10^{-5}$ ft^2/sec, the value of the bed Reynolds number is $\frac{d\sqrt{\tau_o/\rho}}{\nu} = 0.1$. For smaller horizontal velocities the value of the bed Reynolds number will be even less. In fact, the mechanism of scour in a case like this is probably completely different from the usual conception of bed-load movement.

Before the shear is sufficient to cause the material to slide or roll along the bed, the turbulent eddies in the stream will be strong enough to lift the material off the bed in gusts or "puffs." In a wind tunnel with two dimensional flow, Laufer[11] has shown that the root-mean-square value of the vertical turbulent velocity fluctuations near the wall is approximately $\sqrt{\frac{\tau_o}{\rho}}$ for channel Reynolds numbers of 12,300 or 30,800 or 61,600. Since the Reynolds number for the flow in settling tanks is usually of this same order of magnitude, Laufer's results can be applied in a rough way here. In the example above, the settling velocity, v, of a particle of 0.1 mm diameter with specific gravity 1.2 is .0019 ft/sec by Stokes' law at about 20°C. Since $\sqrt{\frac{\tau_o}{\rho}} = .0028$ ft/sec and v = .0019 ft/sec, the eddy currents will certainly be strong enough to lift material off the bed directly. It might be suggested that the velocity should be kept low enough so that $\frac{v}{\sqrt{\tau_o/\rho}}$ is at least greater than 1 or 2.

Another approach to the problem of predicting when scour by turbulent eddies will occur is through the suspended load equation described by Vanoni[12] and others. Neglecting obstacles on the bed (such as scraper blades) and the turbulence caused by them, the relative distribution of suspended material in <u>steady</u> state in a two-dimensional channel is given by the suspended load equation:

$$\frac{C_y}{C_a} = \left(\frac{H-y}{y}\right)^Z \left(\frac{a}{H-a}\right)^Z \qquad \ldots\ldots (11)$$

11. "Some Recent Measurements in a Two-Dimensional Turbulent Channel," by John Laufer, Journal of the Aeronautical Sciences, V. 17(1950)p. 277.
12. "Transportation of Suspended Sediment By Water," by Vito A. Vanoni, <u>Trans.</u> ASCE, Vol. 111 (1946) p. 67.

where H is the total water depth, C_y is the concentration of a given-sized particle at any height y from the bottom, C_a is the concentration at a reference height "a", and Z is defined as

$$Z = \frac{v}{k\sqrt{\tau_o/\rho}} \qquad \frac{2.5v}{\sqrt{\tau_o/\rho}} \qquad \ldots\ldots (12)$$

Here k is the von Karman constant of about 0.4, v is the settling velocity of the specific particle and $\sqrt{\tau_o/\rho}$ is the conventional shear velocity. Although Equation (11) strictly applies only to two-dimensional flow, (as does Camp's turbulence analysis), still it indicates that in any continuous-flow settling tank an amount of light fine material equivalent to the steady state load of the basin will tend to be carried in suspension indefinitely; or more precisely, that sludge may be continuously resuspended by turbulent eddies to maintain this steady state load.

Although Equation (11) is only relative, inasmuch as C_a or $C_a\left(\frac{a}{H-a}\right)^Z$ cannot be determined as yet for various conditions, it is possible to set maximum or safe values for C_a. If the reference point "a" is taken at 0.01 H and if Z = 5, the concentration drops off by a factor of 10^{-5} as y increases from y = 0.01 H to y = 0.09 H (e.g. from 10,000 ppm to 0.1 ppm). If Z = 2, however, the concentration decreases by a factor of only 10^{-2} in the same height (e.g. from 10,000 ppm to 100 ppm). In natural streams, material for which $Z > 2$ is usually not found in suspension in any measurable quantities.

As a safe design criterion, therefore, it is suggested that Z be greater than 3 to 5 or by Equation (12), $\frac{v}{\sqrt{\tau_o/\rho}}$ should be greater than 1.2 to 2. This value checks roughly the value of $\frac{v}{\sqrt{\tau_o/\rho}}$ previously suggested by consideration of the magnitude of the turbulent velocity fluctuations near the bed. The shear velocity in artificial settling tanks is given approximately in relation to the mean horizontal velocity V as

$$\sqrt{\tau_o/\rho} = \frac{V}{18}$$

assuming a Darcy friction factor of .025, and hence $\frac{V}{v}$ should be less than 9 to 15.

The application of this approach can be made clearer by considering an example. Design a settling tank to handle 3.0 mgd and to remove all particles with settling velocities of 0.04 cm/sec. or greater. This corresponds to an overflow rate of 850 gpd per sq. ft. and hence the effective tank surface area should be 3530 sq. ft. To minimize the effect of inlet and outlet disturbances, make the tank long and narrow, i.e. 18 ft. wide and 196 ft. long, and perhaps add 14 ft. to the length to allow for such disturbances.

How deep should the tank be? By Camp's formula for scour, assuming organic particles of specific gravity = 1.2 and diameter = 0.06 mm corresponding to the settling velocity, with conventional values for the constants, it is computed that V_c is at least 7 ft/min. On this basis, the tank need be only 2.2 ft. deep to prevent scour. By analysis with the suspended load equation, however, V should be not more than, say, 12 v_o, or 0.48 cm/sec, equivalent to about 1.0 ft/min. On this basis, the tank should be 15.5 ft. deep, or else a shallower and wider tank could be used.

Referring back to Fig. 9, it may be noted from Camp's discussion of Dobbin's paper,[9] that since

$$\epsilon = \frac{k}{6} H \sqrt{\frac{\tau_o}{\rho}}$$

the abscissa may be rewritten as

$$\frac{vH}{2\epsilon} = 3 \frac{v}{k \sqrt{\tau_o / \rho}} = 3 Z$$

On the basis of the foregoing analysis, then it might be presumed that when $\frac{vH}{2\epsilon} = 3 Z > 15$, the tanks should perform more nearly in accordance with Camp's theory, because scour should be small. However, from an inspection of Fig. 9, it appears that, although there is some relative improvement at higher abscissae, the removal is still too low. This may be due to turbulence in the tank generated by the inlets in addition to turbulence resulting from ordinary shear, because the model tanks are relatively short. Indeed, the criterion given above (i.e. $Z > 5$) may be considered <u>necessary</u> in preventing scour by eddies, but it may not be <u>sufficient</u>, because of the additional sources of turbulence.

Tanks deep enough (or wide enough) to provide low displacement velocities are undesirable because they are costly and they may give rise to density currents that cause short-circuiting. In sewage works, moreover, their increased detention period favors septicity that may affect subsequent treatment adversely. It is desirable, therefore, to develop means whereby tanks can be made shallow and yet the sediment in such shallow high-velocity tanks can be protected from resuspension by turbulent eddies. A suggested design to accomplish this purpose and simultaneously to thicken the sludge is presented in Section VIII of this paper.

V. Multiple Inclined Baffles

A striking illustration of the theories of sedimentation and the ineffectiveness of depth lies in the experiments of Hayden[13] with ore "slimes," containing 2% by weight of granular sand, silt, and colloids in mechanical suspension in water. Hayden fed these slimes over an influent apron into a rectangular tank, 3 ft wide, 9 ft long, and 3 ft deep, with 3 hoppers and a conventional effluent weir. Baffles spaced 3 in. apart and inclined at 45 degrees were inserted in various manners as follows: (a) as longitudinal baffles with the tops at either 4 in. or 0.5 in. below the water surface, (b) as transverse baffles with the tops sloping away from the feed end and either 4.5 in. or 0.5 in. below the water surface, and (c) as transverse baffles with tops sloping toward the feed end and only 0.5 in. below the water surface. The results of Hayden's experiments are presented in Table 1 and illustrated in Fig. 10.

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13. "Concentration of Slimes at Anaconda, Montana," by Ralph Hayden, <u>Trans. Am. Inst. of Mining Engrs.</u> (Montana Volume), Vol. XLVI, (1913) p. 239.

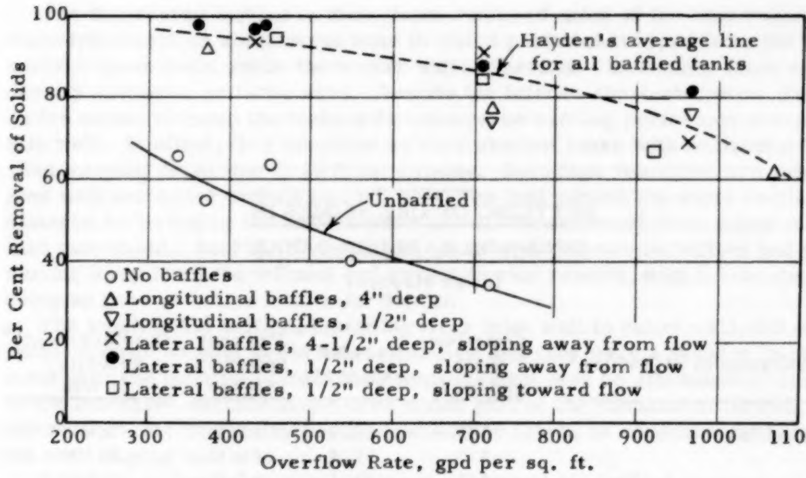
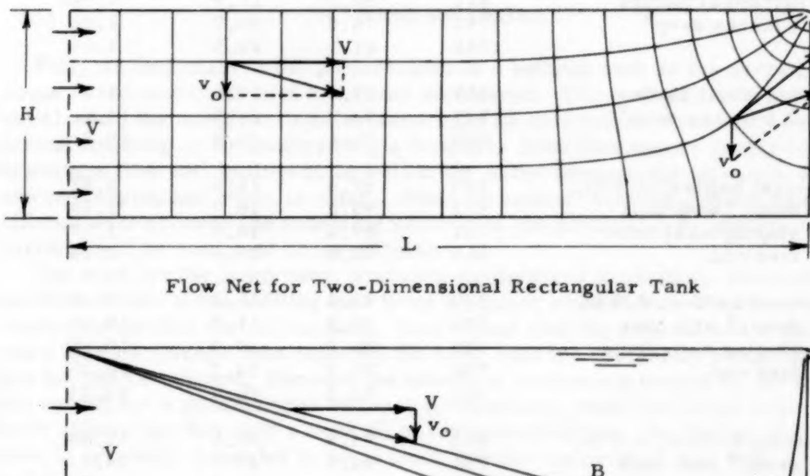


Fig. 10. Effect of Multiple Inclined Baffles on the Settling of Ore Slimes (after Hayden)



Flow Net for Two-Dimensional Rectangular Tank

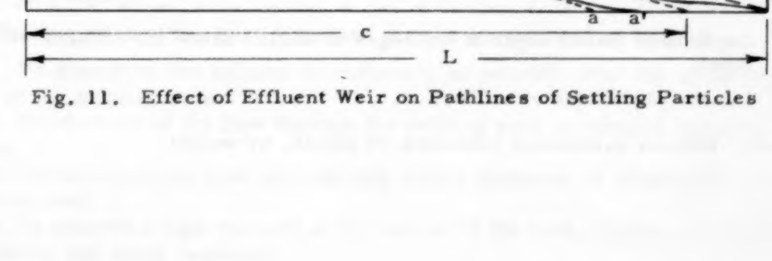


Fig. 11. Effect of Effluent Weir on Pathlines of Settling Particles

TABLE 1

SETTLING OF ORE SLIMES IN OPEN AND BAFFLED TANKS

(after Hayden)

Arrangement of Baffles*	Overflow Rate gpd/sq ft	Percent Removal	Percent Solids in Sludge	Displacement** Velocity, ft/min
No baffles	347	65.9	10.9	0.097
	373	57.4	10.4	0.104
	445	63.1	12.6	0.124
	552	39.7	12.2	0.154
	721	35.3	13.8	0.201
Longitudinal baffles 4 inches deep*	377	92.3	11.9	0.945
	730	76.9	15.7	1.83
	1058	61.1	20.5	2.65
Longitudinal baffles 0.5 inches deep*	589	87.6	17.0	11.8
	729	74.9	24.3	14.6
	960	76.6	25.4	19.2
Lateral baffles, 4.5 in. deep,* with tops sloping away from feed end.	380	97.4	14.2	0.82
	522	93.6	18.7	1.16
	704	89.4	18.8	1.57
	960	70.6	25.5	2.14
Lateral baffles, 0.5 in. deep,* with tops sloping away from feed end.	359	98.5	16.2	7.29
	533	98.3	17.7	10.68
	530	96.0	20.0	10.60
	706	86.8	24.2	14.15
	962	82.3	26.9	19.25
Lateral baffles, 0.5 in. deep,* with tops sloping toward feed end.	533	93.4	28.6	10.68
	704	84.8	21.7	14.1
	925	68.3	27.7	18.5

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* Depth here means depth of top edges of baffles below the effluent weir level.

** In area above tops of baffles.

Note: Influent suspension contained 2% solids, by weight.

The transverse baffles in these tests rendered most of the tank volume ineffective except as a quiescent zone in which particles trapped from the thin surface layer could settle thereafter without serious disturbance from scour, density currents, or turbulence. Despite the intense short-circuiting (based on the entire volume) the tanks with transverse baffling performed exceptionally well. In effect, they operated as very shallow tanks with horizontal velocities varying from one to 20 ft per minute. Such high velocities are associated with increased turbulence, but particles that settled the short vertical distance to the top of the baffles were thereafter protected from scour or bed-load movement. As Fig. 10 indicates, tanks with transverse baffles and tops sloping away from the influent end gave superior results, with 0.5-in. depths being as good or better than 4-in. depths.

The longitudinal baffles, extending from inlet wall to outlet wall, did not perform quite as well as the lateral baffles, but they were a decided improvement over the unbaffled tank. This improvement may be attributable, in part, to the increased surface areas over which part of the suspension flowed. In effect, then, the longitudinal baffles acted like trays, as advocated by Camp,[5] but with sloping surfaces.

Attention is invited particularly to the fact that the baffled tanks of all types gave much denser sludge than the unbaffled tanks. This effect of thickening by inclined baffles is discussed in Section VII.

VI. Inlets and Outlets

Fully as important to the performance of a settling tank as the overall shape is the matter of inlet and outlet structures. The updraft effect of the outlet weir, the simplest to understand, may be ascertained accurately by conformal mapping or by constructing a flow net. Ideal flow theory (required for drawing a flow net) applies quite well to the outlet because the streamlines are converging and shear is small. When the updraft velocity approaching the effluent weir exceeds the subsiding velocity of the particle to be settled, the particle will be entrained in the effluent.

The inlet, on the other hand, produces anything but ideal flow. Invariably the flow comes to the settling tank from a pipe or channel with a much smaller cross section than that of the tank. This means that the streamlines must in some fashion diverge upon entering the tank, with the customary result of separation and turbulence. Because the effects of turbulence caused by the inlet can extend for a considerable distance downstream, while the outlet weir updraft affects the flow only a very short distance upstream, the design of the inlet is generally conceded to be of much greater importance than that of the outlet.

Inlet Structure

The purposes of the inlet structure may be enumerated as follows:

1. To distribute the influent as uniformly as possible over the cross section of the settling zone,
2. To start all of the flow through the settling zone in parallel horizontal paths,
3. To introduce the flow into the tank with a minimum of large-scale turbulence, and
4. To prevent a high velocity at the bottom of the tank, where sediment is gathering and being removed.

From the foregoing statement of purposes it is evident that the ideal inlet would be a diffusion wall, with openings large enough to pass all of the suspension, across which there would be a head loss great enough to insure uniform velocity distribution over the cross section of the settling zone. The ideal inlet moreover, would have expanding passageways on the downstream side so that, as nearly as possible, the kinetic energy of the influent would either be converted to potential energy or dissipated within or near the inlet wall, without becoming a disturbing influence in the main body of the tank.

In contrast to the ideal diffusion-wall inlet, consider a simple free-fall weir, a poor type of inlet structure. The inlet weir achieves a good transverse flow distribution, but beyond this its virtues cease. The vertical velocity distribution just downstream from the weir is likely to show a strong flow along the bottom with reverse flow at the top, indicating a large eddy. The weir introduces kinetic energy in the amount of the free fall, but it absorbs practically none.

Between these two extremes lie the practical possibilities of inlets for settling tanks. The new design of oil-refinery waste-water separators utilizes a vertical slotted baffle with slots diverging at a 15° angle to approximate the ideal diffusion wall. The free area at the 5/16" throat of the slots is only 4% of the cross-sectional area of the tank. The kinetic energy at the throat is therefore 625 times that in the tank. Before the flow leaves the diverging slots, however, its velocity is reduced to about 3 times the displacement velocity in the tank. Tests of the head loss across such a baffle have shown that the recovery of energy in the diverging slots is nil. Therefore, the baffle converts about 98% of the kinetic energy existing at the throat into small-scale turbulence, which is quickly dissipated.

In sewage settling tanks this slotted baffle is not practical because it is difficult to keep clean. Workable inlets to sewage tanks generally employ entrance ports or pipes which at least distribute the influent across the width of the tank. Various schemes ranging from a simple tee on the end of each inlet pipe to the sloping target baffle used at Detroit are employed to deflect the jets from the inlet ports. All such inlets involve considerable large-scale turbulence; but if the tank is made long and narrow, the inlet disturbance is confined to a relatively small part of the tank.

Outlet Structure

The entraining effect of the outlet weir has been analyzed approximately in a National Research Council report.[14] There it is shown that a certain portion of the tank volume, in the shape of a cylindrical sector with its axis at the outlet weir, is ineffective for settling because a particle entering this zone will be entrained in the effluent. The equation given for this volume is

$$\text{Ineffective volume} = k\, q^2/v^2 \qquad \ldots\ldots (13)$$

where q is the weir loading or flow per unit length of weir, v is the settling velocity of the particle considered, and k is a constant ranging from 0.35 to 0.55 depending on the shape and geometry of the tank.

This analysis, based on ineffective volume, does not give a clear comparison with ideal tank theory, which assumes a uniform vertical distribution of

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14. "Sewage Treatment at Military Installations." Report of the Subcommittee on Sewage Treatment, NRC, Sewage Works Journal, Vol. 18, (1946) p. 791.

velocity at the outlet of the settling zone. Particles which have not actually reached the bottom are entrained in the effluent from the ideal tank and thus, according to this reasoning, even a part of the volume of an ideal tank is ineffective in settling.

A more proper examination of the effect of the outlet weir can be made by sketching the flow net for the tank to be studied. This approach indicates the effect of the weir on the velocities through the tank and not just in the vicinity of the weir. Consider three particles, A, B, and C all starting at the top of the tank at the inlet. The velocity of each particle at any point is given by the vector sum of its settling velocity and the local water velocity. From such velocity vectors, the paths of the three particles have been traced on Fig. 11.

Particle A, with settling velocity v_A, would reach the bottom at distance a from the inlet in an ideal tank. By means of the flow net it may be determined that, with the effect of the outlet weir, A will not reach the bottom until the distance a'. Particle B, which would have just settled to the bottom in the length L, of the ideal tank, will be entrained if the updraft velocity is greater than its settling velocity. Particle C, which may be called the critical particle, would settle in an ideal tank at length c from the inlet. With the weir updraft, however, particle C is carried along toward the weir and finally just escapes the strong updraft and settles to the bottom at the end of the tank. All particles with $v > v_C$ are settled completely, and those with $v < v_C$ will be entrained in the effluent if they start at the top of the tank. The effect of the weir, then, is to decrease the length of equivalent tank from L to L'. To find this accurately, a different flow net must be drawn for each value of L/H desired. It will soon be found, however, that for long tanks, the distortion of the flow is negligible at a distance of a few times the depth. Hence to minimize outlet effect, a long tank is again desired. It is common practice, moreover, to introduce additional launders or take-off weirs to keep the weir rate down.

The relative importance of inlet designs on the hydraulic characteristics of a tank is illustrated graphically in Fig. 12, which is a set of dimensionless dispersion curves for different inlet conditions on a 10 x 5 x 2 ft tank, all at the same flow rate.[10] The simple weir inlet is seen to produce a dispersion curve not greatly different from that of an ideal mixing tank, indicating the ineffectiveness of this inlet arrangement. A reversed-flow weir inlet, followed by passing the influent through a tray of raschig rings, produces a dispersion curve with a much higher peak, indicating less mixing, but the short-circuiting is still severe. The diverging slot baffle inlet, mentioned before, produces the best curve of the three inlets with the greatest median and modal detention times.

With the slotted baffle for an inlet, the outlet structure was varied in at least three different ways, but no significant difference could be observed from the results (not shown in Fig. 12). Insofar as dispersion characteristics are concerned, differences in outlet are of small importance compared to differences in inlet.

VII. Thickening

Particles that settle through zones of free and/or hindered settling form a carpet of sludge when they reach a flat or inclined surface in a tank. This carpet, in its upper portion and in early stages of formation, consists of particles resting upon each other in a haphazard fashion with a high proportion of interstitial space, i.e. a high porosity and a high void ratio. Flocculent

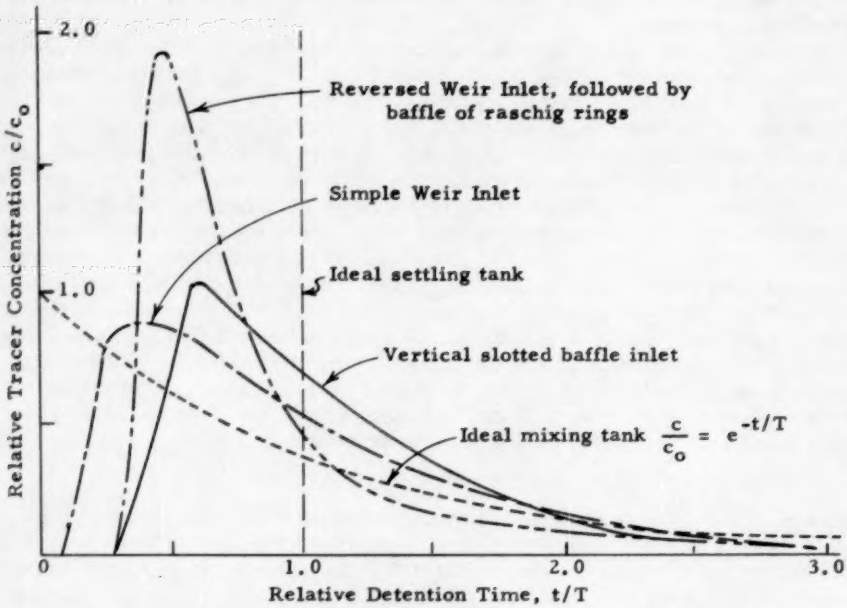
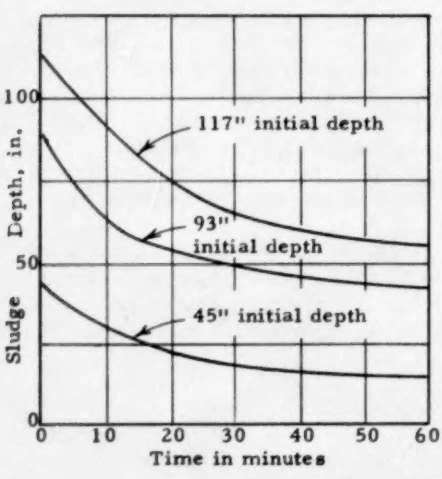


Fig. 12. Dispersion Curves for 10' x 5' x 2' Tank With Various Inlets (Horizontal Velocity = 1.5 ft/min)



Initial Depth	Depth of Sludge in Per Cent of Initial Depth After 1 hr.	3 hrs.	19 hrs.
117"	46	37	12
93"	40	31	14
45"	32	22	16

Fig. 13. Compacting of Activated Sludge (After Rudolfs and Lacy)

particles, such as activated sludge, form carpets or blankets with extremely high porosity.

Subsequent to its formation, the sludge blanket is subjected to excess hydrostatic pressure from its own weight. The blanket tends to compact as water is squeezed from the interstices by increased weight or by mechanical modification of the honeycombed structure. In effect, then, thickening is the process of removing liquid from the solids; or the opposite from sedimentation, which is the removal of suspended solids from the liquid.

Thickening is generally a desirable phenomenon in settling tanks from which the sludge must be pumped and otherwise processed. A sludge with 98% moisture has approximately twice the volume per unit weight of dry solids as a sludge of 96% moisture. Denser sludges yield savings in pumping costs, pipelines, and digestion tanks, but a sludge that is too dense may become troublesome to handle.

Factors that influence the initial structure and the thickening of sludge blankets include:

1. The nature of the settleable solids, i.e. their density, shape, flocculent structure, thixotropy, content of living organisms, electrostatic charges, and other surface characteristics.
2. The concentration of settleable solids in the original suspension.
3. The dissolved substances in the interstitial water.
4. Temperature.
5. The depth of the blanket.
6. The surface area of the blanket.
7. The time allowed for compaction.
8. Structural modification by mechanical action, vibrations, or pressure.

Of these factors, the first three are independent of the design of the settling tank and the fourth is relatively so, although shallow tanks may heat or cool more rapidly than deep ones. The other factors, closely related to the shape and design of settling tanks, are discussed below.

Effect of Depth, Surface Area, and Time

The initial compaction of a sludge blanket is analogous to consolidation of clay soil and to the cooling of a hot solid body, all of which phenomena can be formulated by the same partial differential equation. For one-dimensional consolidation of a clay soil, water in the inner regions cannot be squeezed out rapidly, for it must pass upward successively through countless other interstices and capillaries as hydraulic and physical considerations permit. When the amount of compaction is small, the partial differential equation governing this phenomenon is:

$$\frac{\partial u}{\partial t} = C_v \frac{\partial^2 u}{\partial z^2} \quad \text{. . . . . (14)}$$

where u is the excess pore pressure, i.e. any pressure in excess of the hydrostatic pressure, and C_v is the coefficient of consolidation. When an external pressure is applied to a clay soil, the pressure is initially carried to a very large extent by the interstitial water, creating an excess hydrostatic pressure, u. Gradually as water is squeezed from the upper layers of clay the excess pressure is transferred to the grain structure. The reduction of pore pressure near the surface creates a pressure gradient which causes the pore water to

flow upward. The time required to reach a given degree of average consolidation in a clay layer is proportional to the square of its thickness. Hence, a shallow bed of clay will consolidate much more quickly than a deep bed or, to express it otherwise, after a given interval of time a shallow bed will have achieved percentagewise more consolidation than a deep bed.

The early stages of compaction of a sludge blanket in a settling tank are roughly analogous to the consolidation of clay under an external pressure, and hence a shallow blanket of sludge thickens more rapidly than does a deep blanket. The analogy is approximate, however, inasmuch as (a) the amount of subsidence is large compared with the initial thickness and (b) the excess hydrostatic pressure in the case of a sludge blanket arises from the weight of the sludge itself rather than from an applied force. As more sludge accumulates, the weight increases. Moreover, as compaction of a given blanket progresses, the excess force on lower layers increases as more and more load is transferred to the grain structure.

The coefficient of consolidation in equation (14) is defined as

$$C_v = \frac{k\,(1 + e)}{a_v\,\gamma} \qquad \ldots\ldots (15)$$

where k is the coefficient of permeability, e is the void ratio, a_v is the negative slope of the pressure-void ratio curve, and γ is the specific weight of water. In the consolidation of clay, the components of C_v do not change markedly and hence C_v remains approximately constant. In the compaction of sludge, however, both k and e in the numerator decrease rapidly as thickening progresses but a_v also decreases as the pressure increases.

The effect of depth on the compacting of activated sludge is illustrated by Fig. 13, from data presented originally by Rudolfs and Lacy.[15] Identical sludges were placed in cylinders of three different depths and the rates at which the sludge surfaces dropped were noted. The three curves are roughly parallel for the first hour, by which time the shallowest sample had thickened to 32% of its original volume and the deepest one to only 46%. During the very early stages of compaction, the bottom of the blanket has practically no effect inasmuch as compaction is occurring primarily at the surface. Hence, in absolute distance the top of a thin layer subsides almost as rapidly as the top of a thick layer and consequently for a thin layer the relative amount of compaction will be larger. The advantage of the shallowest sample was still maintained after three hours, but by 19 hours the deep sample had formed the densest sludge. For non-putrescible sludges long periods of compaction are beneficial, but in sewage works biological considerations require that sludge be removed from settling tanks in one or two hours at the most.

It is apparent, therefore, that the shallowest sludges thicken most rapidly and consequently for a given volume of sludge the greatest possible area should be provided. Thus, the simple initial compaction of sludge, like the settling of discrete particles, is a function of surface area of the tank. After the first hour of such compaction, the depth of the sludge blanket becomes an important consideration. In water works, where alum or calcium-carbonate sludge may be allowed to accumulate for several days or weeks, additional depth for sludge should logically be provided.

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15. "Settling and Compacting of Activated Sludge" by Willem Rudolfs and I. O. Lacy, Sewage Works Journal, Vol. VI, No. 4, 1934, p. 647.

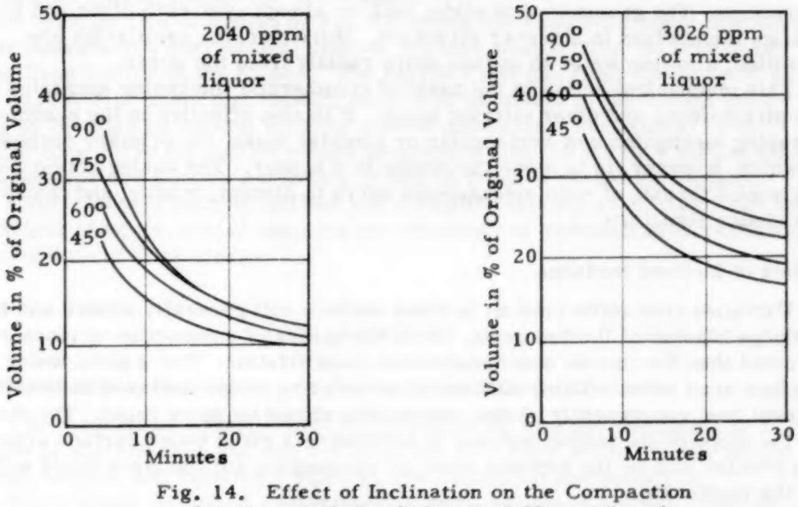

Fig. 14. Effect of Inclination on the Compaction of Activated Sludge (After Rudolfs and Lacy)

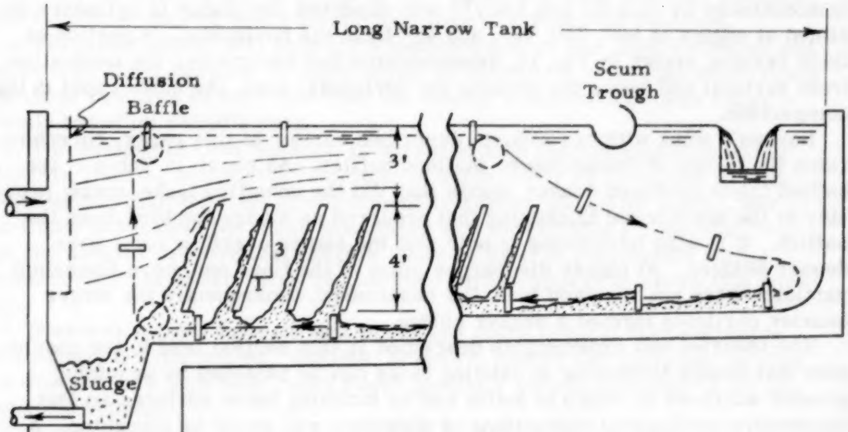

Fig. 15. Tentative Suggested Design of a Baffled Tank for Water or Sewage Works

Effect of Mechanical Action and Vibrations

The honeycombed structure of sludge formed by discrete as well as flocculent particles is extremely unstable when subjected to lateral forces or vibrations. The grains tend to slide, roll, or slough over each other and to realign themselves in a denser structure. Moreover, the capillaries are modified, allowing water to escape more rapidly from the pores.

This mechanical action is the basis of commercial thickening apparatus of the picket-fence and other stirring types. It is also effective in the sludge-scraping mechanisms of rectangular or circular tanks, the primary purpose of which, however, is to move the sludge to a hopper. The raking action and horizontal thrusts of such mechanisms serve to disrupt, modify, and thicken the sludge structure.

Effect of Inclined Surfaces

Particles that settle onto an inclined surface will generally adhere and form a sludge blanket of limited depth. Here the theory of compaction is more complicated than for simple one-dimensional consolidation. For a given water surface area more settling surface is provided by an inclined bottom than by a level one; consequently sludge compaction should be more rapid. The steeper the slope of the bottom surface in relation to a given water surface area the greater will be the exposed area for compaction and the more rapid will be the thickening.

On steep slopes, moreover, sludge will slough or flow downward, rather than accumulate as a deep blanket. This sloughing or tumbling action is similar in effect to mechanical action or vibrations in breaking up the flocculent honeycombed structure of the sludge and causing the individual grains to realign themselves in a denser structure.

The effect of various slopes on the compaction of activated sludge was also demonstrated by Rudolfs and Lacy[15] who observed the sludge in cylinders inclined at angles of 90°, 75°, 60°, and 45° from the horizontal. A portion of their results, shown in Fig. 14, demonstrates that the greater the inclination from vertical and hence the greater the horizontal area, the more rapid is the compaction.

Hayden's work with ore slimes, described earlier in this paper, corroborates the theory of thickening by inclined baffles. As shown in Table 1, the baffled tanks produced denser sludge than did the unbaffled tanks, owing probably to the mechanical thickening that occurred as sludge tumbled down the baffles. It is also interesting to note that the higher overflow rates gave denser sludges. At higher discharges many of the finer and more flocculent particles were not removed from the suspension; consequently the larger heavier particles formed a denser sludge.

The theories and experiments described in this section lead to the conclusion that sludge thickening in settling tanks can be hastened by providing greater surfaces on which to settle and by inclining these surfaces so that successive mechanical disruptions of structure will occur by sloughing. A suggested design for such a tank is presented with the following section.

VIII. Suggested Design

A suggested design of a settling tank for water or sewage works, with a tentative arrangement of multiple inclined baffles, is shown in Fig. 15. For flocculent putrescible wastes, inclined baffles cannot be spaced too closely

for fear of clogging. Inclinations should be greater than 45 degrees to prevent heavy accumulations of sludge and to assure rapid sloughing. Tentatively, therefore, it is suggested that baffles be inclined on a slope such as 3 on 1 and that they be spaced one foot or more apart.

The arrangement of baffles in Fig. 15 allows the sludge-scraping mechanism to operate in a conventional manner to move sludge from the bottom of the inclined baffles to the hopper. The return flight can skim the surface, if desired, but it is likely that floating scum will be moved to the trough effectively by the high horizontal velocity without mechanical assistance. The settling zone above the baffles should be greater than the 0.5 or 4 inches used by Hayden for ore slimes, but depths of two or three ft should be adequate. This combination of inclined baffles and a shallow settling zone, free from the probability of scour, should improve the efficiency of sedimentation and favor the formation of thick sludge.

IX. SUMMARY

The major points of this paper may be summarized as follows:

1. The theories of Hazen, Camp, et al, indicating that sedimentation is a function of surface area and that for optimum efficiency settling tanks should be long, narrow, and relatively shallow, are reiterated and supported.
2. The present method of comparing the performance of settling tanks on the basis of their total percentage removal of suspended solids is inadequate. Instead, a new method is proposed to compare the performance of an actual tank with an ideal basin by means of the "overflow residual efficiency."
3. Experimental results with wax spheres and silica verify to a large extent the theory that sedimentation is independent of depth, but these tests indicate that turbulence and/or scour at very shallow depths and high displacement velocities may be more deleterious than was formerly realized.
4. Turbulent eddies will produce a resuspension of fine light sediment and lead to inefficient removal of these solids at horizontal velocities much lower than those required to start bed-load movement in accordance with Camp's formula based on Shield's data.
5. The suspended load equation is advanced as a logical approach to the subject of limiting horizontal velocities to avoid scour by turbulent eddies. This equation indicates that displacement velocities should be much lower than Camp advocates; but since deep tanks are undesirable for several reasons the authors propose that shallow tanks be used in conjunction with a method to prevent scour. Multiple inclined baffles appear to serve this purpose.
6. Dispersion tests indicate that inlets are more critical than outlets in controlling short-circuiting. Outlets, however, modify the flow nets, so that poor designs with high weir rates cause partially settled particles to be entrapped in updrafts.
7. A given volume of sludge will thicken more rapidly during early stages of compaction in a shallow blanket than in a deep one; hence a large bottom area should be provided for sludge thickening.
8. Thickening is accelerated by inclined surfaces and by mechanical action or vibrations.

On the basis of the foregoing analysis, it is suggested that further research and experimentation be conducted with a view toward designing rectangular

settling tanks for water and sewage works along the following lines, (a) with a surface area corresponding to the overflow rate of the particle to be 100% removed, (b) with high ratios (e.g. 10) of length to width in order to minimize inlet and outlet disturbances, (c) with depths great enough to accommodate mechanical scraping mechanisms (e.g. 6 ft or more) and (d) with multiple inclined baffles to minimize scour and to improve the thickening of sludge.

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c. Discussion of several papers, grouped by Divisions.

d. Presented at the Atlanta (Ga.) Convention of the Society in February, 1954.

e. Presented at the Atlantic City (N.J.) Convention in June, 1954.

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